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SAND COMPACTION BY VIBROFLOTATION

by Elio D'Appolonia, Callix E. Miller, Jr.,
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SAND COMPACTION BY VIBROFLOTATION

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Introduction

Vibration and sudden shocks are known to change a sand deposit from a loose to a dense state. This change is accompanied by surface settlements that approach one inch per foot of depth of sand deposit. To avoid the severe damage that would result to a structure from such settlements, it is necessary to compact the sand prior to construction or to by-pass the loose sand deposit with piles.

Several methods of sand compaction have been used - driving of short stubby piles, blasting[1*], rubber tire rollers[2], vibrating plates and vibroflotation[3,4,5] - the method selected depending on its effectiveness and economy. Cost comparisons showed that at the site of the new twelve-million dollar phosphate plant of the International Mineral and Chemical Corporation, near Bartow, Florida, the vibroflotation process was the most economical. A comparison of the effectiveness and positiveness of each of the above methods likewise resulted in the selection of this process. The method is simple and with field control good compaction is assured.

While vibroflotation has been used extensively in Europe, notably in Germany[6] it has hitherto found little application in the United States. The first large scale compaction of loose sand by vibroflotation in this country is described herein. Approximately 156,000 square feet of foundation area were compacted to an average depth of 12 feet at a cost of approximately $0.10 per cubic foot. Compaction proceeded on an average of 1400 square feet per day, requiring 25 to 30 penetrations of the vibroflot during each eight hour shift.

It is the purpose of this paper to describe the soil conditions and foundation loads that led to the selection of the vibroflotation process and to report on the results of the field and laboratory tests conducted to establish the effectiveness of the process.

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1 Bibliography appended.

Soils

The site of the phosphate plant is a relatively high rise of land with a gradual fall to the west. The high point of the site is at elevation 190 and the low point at 160. Examination of aerial maps disclosed a uniformity in vegetation throughout the area. There were no thick rings of vegetation indicating the existence of old swamps. Borings for soil exploration were located to check this apparent uniformity of the soils and to disclose soil conditions immediately beneath the major structures. Fig. 1 shows these buildings and location of the borings. A generalized boring log with the results of the standard penetration tests (the number of blows required to produce one foot of penetration of a two-inch split spoon under a weight of 140 pounds falling through a height of 30 inches) is shown in Fig. 2. Soil profiles Figs. 3 and 4, show an area covered with clean, loose, wind-blown sand varying in thickness from 10 to 15 feet. Penetration tests disclose that the density increases with the depth of the sand deposit, the number of blows ranging from 2 at the surface to 10 at the bottom of the layer. Such low values indicate the presence of a very loose sand that is susceptible to large volume changes when subjected to vibration or sudden shock.

Below the sand is a sandy "matrix" deposit containing reddish brown and yellow clay and some phosphate granules. (Matrix as used here is a local term that denotes a stiff conglomerate of sand, clay and phosphate granules.) The average thickness of this layer is 15 feet. Penetration results reveal a material that is medium dense or fairly well compacted. The matrix has high bearing strength and vibrations would have very little effect on this deposit.

Beneath the matrix are layers of sandy and gravelly clay interspersed with layers of sand and matrix. The clay deposits vary in thickness from a few inches to 3 feet. For the most part the sandy clay deposits are firmly cemented as indicated by the penetration tests and would be classified as medium stiff to stiff. The reddish brown to yellow discoloration of the originally white clay above elevation 135 is likely caused by exposure to air and subsequent desiccation.

The clay deposits below elevation 135 contain a great deal of coarse gravel, rocks and traces of phosphate granules. Penetration tests are erratic indicating deposits of soft stiff material with sporadic refusal near elevation 120. Because of the rocks and boulders the results of the penetration tests below elevation 120 have to be interpreted with caution. Resistance to penetration of a comparable magnitude may not be encountered during the driving of piles since the rocks would be easily pushed aside. Observations of deep cuts made by mining operations in similar soils disclose alternate layers of stiff and soft clay matrix erratic in extent and depth. Such deposits could very easily be classified by a driller, and erroneously so, as gravelly clay. Hence the resistance to penetration below elevation 120 as recorded by the driller, could not be relied upon for design of the foundations. Below elevation 130 the gravelly clay layer has a blue to black discoloration because of pre-existing vegetation at these lower elevations.

Soil Tests

Mechanical analysis tests conducted on samples taken from different depths of the upper sand deposit indicate a very uniform sand, see Fig. 5. In all cases the grain sizes are greater than 0.06 mm. These tests clearly indicate a wind-blown sand which in a loose state is unstable.

Table 1 gives the results of tests conducted on undisturbed clay samples. The location of each sample is shown in Figs. 3 and 4. The high moisture contents and liquid limits obtained for the clay samples would normally indicate a fairly compressible material. However this is not borne out by the compression indices obtained. The values are low indicating a material of low to medium compressibility, which probably can be attributed to the precompression by desiccation that occurred during the formation of the deposits.

Field density tests conducted at various points throughout the foundation area and at different depths below the surface to a maximum of 12 feet disclose a loose sand with a relative density ranging from 30% to 47% and averaging 33%. The sand had an average dry unit weight of 90 and 110 pcf in its loosest and densest state respectively. The dry unit weight of sand in a dense state was obtained by the vibration of a saturated sample under a surcharge of 2.2 ksf. The minimum dry unit weight was obtained by tremming the sand through a funnel.

The maximum dry unit weight depends on the test procedure. Different methods of compaction produced variations in the dry unit weight from 106 to 114 pcf. The question arose as to whether or not the average value of 110 pcf was reasonable. The results of the above method of compaction were compared with modified Proctor tests[2] conducted on the same sand. Fig. 6 shows that 100% modified Proctor corresponds approximately to 100% relative density at a dry unit weight of 110 pcf. Because of this agreement it was felt that field control of the vibroflotation process based on maximum and minimum values of dry unit of 110 and 90 pcf was adequate.

Minimum dry unit weights were obtained using different procedures of pouring sand into the container. Results agreed within 1.5% of the average value of 90 pcf.

The specific gravity of sand is 2.67.

Foundations

A study of the soil explorations and laboratory data indicates that settlement of the foundations may be the result of two well-defined causes:

(1) A change in state of the upper layer of sand from loose to dense.

(2) Consolidation of the different layers of clay below elevation 150, approximately 30 feet below grade.

Settlement because of change in state of the loose sand may result from vibrations caused by railroad operations and the vibrations set up by the machinery, mainly the battery of sixteen 300 HP motors and vacuum pumps in the process building.

Settlement because of the consolidation of the clay layers depends on the pressure imposed upon them by the various foundations. Since the footings for most of the main buildings will have a width of 10 feet - less than one-third of the depth to the compressible clay layers - and will be subjected to light loads, settlement due to consolidation will be negligible. For example, the average uniform pressure imposed by the process building is 500 lbs per sq.ft. and the largest footing is 10 feet square. Load conditions are similar for other buildings with the exception of the two main storage buildings. One is a three-hinged arched structure with a 200 foot span and 600 feet long, having an average floor load of 1200 lbs per sq. ft. The second is a 4-bin silo, 110 feet high, supported on a rigid raft foundation approximately 100 feet square. Dead load is 1100 lbs per sq. ft. and dead plus live (full) load is 3300 lbs per sq. ft.

The design of successful spread footings for all but the two structures mentioned above, requires a compaction of the loose sand and the base of the footings kept as near the ground surface as possible.

Piles 50 to 60 feet long, with tips reaching to the gravelly clay layer, approximate elevation 120 were considered for the foundation of the storage buildings. Investigations of the foundation for the silo storage, showed that the settlement of a pile foundation would exceed that of a raft foundation placed on a compacted sand. Even though standard penetration tests indicate high resistance and refusal at elevation 120, it is not certain that this resistance would be obtained during the driving of piles. As pointed out earlier these lower deposits are a conglomerate of hard and soft clay formations. Hence the piles would act as friction rather than bearing piles. The pressure imposed on the clay layers by the frictional forces of the piles would result in consolidation of the clay layers and surface settlement. The pile load would in turn be gradually transferred to the tip of the piles and the increased pressure would result in further settlement.

Short point-bearing piles driven to refusal into the sandy matrix to bypass the loose sand deposits offer no advantage. The transfer of load to the pile tips imposes a greater soil pressure on the underlying clay layers than spread footings near the surface and hence larger settlements would occur. Apparently it would be difficult to drive piles to elevation 120 and to penetrate the sandy matrix without "jetting".

It was concluded that successful spread footings and raft foundations could be designed provided the loose sand was compacted. To compact the sand the vibroflotation process was selected. In this process[3] a 15-inch diameter tube is vibrated by a rotating eccentric weight mounted within the tube. The vibration and movements of the sand particles produced by the vibroflot are sufficient under saturated conditions for adequate compaction of the sand within a radius of 4 feet.

Preliminary Compaction Tests

As previously mentioned, the dry unit weight of Bartow sands ranged from a minimum of 90 pcf to a maximum of 110 pcf (obtained by vibration of the container). The specific gravity of the sand is 2.67. From the definition of relative density

$$D = \frac{e_L - e_N}{e_L - e_D} \qquad (1)$$

it can be shown that

$$D = 5.50 - \frac{495}{\gamma N} \qquad (2)$$

where

e_L = loose void ratio
e_D = dense void ratio
e_N = natural void ratio
γN = natural dry unit weight in pcf

A sand with a relative density of 70% or greater was selected as the criterion for adequate compaction. This selection was based on experience and statements of previous investigators[5,7]. A sand with a relative density greater than 70% would be suitable foundation material. Settlements due to the densification of the sand from vibrations and sudden shocks would be negligible. Hence the compaction process has to be so conducted as to increase the dry

unit weight of the soil to a minimum of 103 pcf. To do this by vibroflotation, the most effective pattern and spacing of the vibroflot had to be established.

Three patterns were studied - the line, square and triangle - with different spacings between points of compaction. Tables 2 to 5 show the results of the tests. In each pattern the vibroflot was withdrawn (after maximum compaction had been obtained) in one foot lifts. The sand added to each hole averaged 2-1/4 cubic yards.

A 6 inch Shelby tube (Fig. 7) equipped with a driving head was used for the field density tests. The points - with relation to the vibroflot compactions - at which the tests were made are shown in the diagram on each of Tables 2 to 5. Each compacted area was excavated and field density tests were made at depths 4, 6 and 8 feet below the ground surface. Each of the relative density values given in the tables is an average of at least three tests. For pattern 2 shown in Table 3 the relative density at point 1 is an average of 12 tests.

It is reasonable to assume that the effectiveness of the process will decrease as one moves away from the center of the compaction. The effect will at first decrease rapidly and then more slowly as the distance from the vibroflot increases - an exponential type of variation. A plot of the test results, Fig. 8, shows this variation.

With the smaller spacing, pattern 4, the greatest increase in density was obtained. The overlapping effect of each vibroflot is pronounced and relative densities of 70% or greater are obtained throughout the whole of the compacted area.

The results of the line pattern with a 7'-4" spacing between compactions and the square and triangular patterns with 8'-0" spacings are nearly the same. For distances 2'-6" and less from the center of the vibroflot, shorter spacings improve the compaction. At distances greater than 2'-6" the effects of changes in pattern and spacing are not noticeable.

From these preliminary tests it is concluded that:

1. The compaction of a single vibroflot does not increase the relative density above 70% at points greater than 3 feet from the vibroflot.
2. The overlapping effect of the vibroflot spaced further than 8 feet apart is small.
3. The effect of compactions can be superimposed.
4. The spacing of the vibroflot at 6 feet or less gives relative densities of 70% or greater throughout the compacted area.

Though not conclusive, the difference between a square or triangular pattern is not significant. The triangular pattern is to be preferred because it has the greatest amount of overlapping of compaction effort.

Design

From the foregoing results a procedure for determining a suitable spacing of the vibroflot was developed. Fig. 8 shows how the compaction decreases with distance from the vibroflot. An arbitrary measure is needed to describe this variation. Conveniently then, a number or an influence coefficient is used to denote the degree of compaction at each one foot interval from the center of a single vibroflot compaction. The dotted curve of Fig. 8 is taken as the compaction due to a single vibroflot. It should, as it does, follow the lower field curve.

Arbitrarily the influence coefficient at a distance of 6 feet from the vibroflot is chosen as one. The coefficient at each one foot interval towards the vibroflot is then scaled as the ordinate to the dotted curve of Fig. 8. The plot of the influence coefficient against distance from the center of the vibroflot is

shown in Fig. 9a. From Fig. 8 one can also readily plot the relationship between influence coefficient and relative density.

For design it is then assumed that if - at any point in a pattern of vibroflot compactions - the sum of the influence coefficients is greater than 10 a relative density of 70% is obtained.

To check this assumption a triangular pattern with 7'-6" spacings as shown in Fig. 10 was tested. From the design curves of Fig. 9 the sum of the influence coefficients at the critical point A is 12. A relative density greater than 70% should be found throughout the area enclosed by the sides of the triangle.

The results of the field density tests conducted in an excavated pit at 3, 6, and 9 feet below ground surface are summarized in Table 6. The field data are in good agreement with the design curves of Fig. 9.

Field Tests

Compaction at the Bartow site was conducted in a triangular pattern with 7'-6" spacings of the vibroflot. Approximately 3.5 acres (shown as shaded in Fig. 1) were compacted to an average depth of 12 feet.

Field control on the compaction was maintained by density tests taken in 10 test pits dug to an average depth of 10 feet. Tests were conducted at 3 foot intervals from the ground surface. In addition field density tests were made at the bottom of the excavation of nearly all major footings. Fig. 11 shows the results of random field density tests made at the bottom of the excavations for the footings of the process building. The soil was tested at points of weakest compaction. The curves shown in Fig. 11 are typical of the test results obtained from the other foundation areas. A total of 142 field density tests were made to check the sand compaction. Fig. 12 shows the wall of a 9 foot deep excavation in a compacted area. The white regions denote the core of compacted white sand which was added. It is interesting to note that the surcharge of a crane located approximately 5 feet from the edge of the cut has not caused a failure of the vertical sand face.

Settlement

Bench marks placed solidly in the soil were located throughout the area. Settlement readings of the walls and footings of the various buildings were read during and after construction. The data presented are for the large silo storage bin. This structure has a raft foundation approximately 100' x 100'. Placed on this mat are circular bins 40 feet in diameter. The average uniform dead load is 1,100 psf. The maximum dead plus live load is 3,300 psf.

Because of its function, it is quite likely that the bins will be full about three-fourths of the time. Hence settlement calculations based on the above two values of uniform pressure made to "box-in" the estimated settlement.

Boring STS-B3, Fig. 3, shows the compressible layers beneath the silo storage building. It was estimated that the total thickness of compressible material between elevation 152 and 118 was 15 feet. The remainder was neglected in the settlement computations. The boring showed that the clay and sand deposits were in alternate layers one to two inches thick. Soil explorations also indicated a clay layer 3 feet thick between elevation 152 and 148. The undisturbed 3-inch Shelby tube soil sample 12 inches thick removed from elevation 150 was a gray clay with soil properties as shown in Table 1 under soil sample F3-4. It was assumed that this and similar samples from the other borings were representative of the 15 feet of compressible soil.

The results of classification and consolidation tests are shown in Table 1. Based on these tests an average value of

$$\frac{C_c}{1 + e_o} = 0.103$$

was used in the settlement computations.

The estimate of the dead load settlement was 1.43 inches and the dead plus full live load was 3.96 inches.

If the compressible clay layer were continuous and had drainage top and bottom approximately 650 days would be required for 90% consolidation. However, because of the alternate layers of clay and sand it is estimated that roughly 200 days would be required for 90% consolidation.

The calculated settlement-time curves for dead load and dead plus live load are shown in Fig. 13.

Settlement readings were taken at 12 points on the periphery of the foundation. The record was started in May just prior to the construction of the walls of the bins. The dotted curve of Fig. 13 shows the average of the 12 points of observation extending over a period of 131 days. The observed settlement is less than the calculated settlement for dead load by approximately 50%.

Settlement observations will be continued for a period of several years to determine the movement of the building due to changes in load and to the level of the water table during the rainy season.

Summary

The loose Bartow sands were adequately compacted with a vibroflot spaced at 7'-6" in a triangular pattern. Relative densities of 70% were found at points of weakest compaction and the effectiveness of the vibroflotation process was established. Design curves for compaction of clean, uniform sands similar to those in the vicinity of Bartow, Florida, were developed.

Settlement records were obtained for the different buildings during and after construction. Observations of the movement of the foundation for the silo storage building show a settlement 50% less than the calculated dead load settlement.

Credits

The Rust Engineering Company of Pittsburgh, Pennsylvania, was the engineer and constructor for the phosphate plant. Mr. S. Steuerman, owner and operator of the vibroflot carried out the sand compaction.

The writers are indebted to Mr. J. Kelvington of the Rust Engineering Company for reviewing the field data and preparing the drawings.

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7. D. M. Burmister, "The Importance and Practical Use of Relative Density in Soil Mechanics", ASTM, Vol. 48, 1948, p 1249-1268.

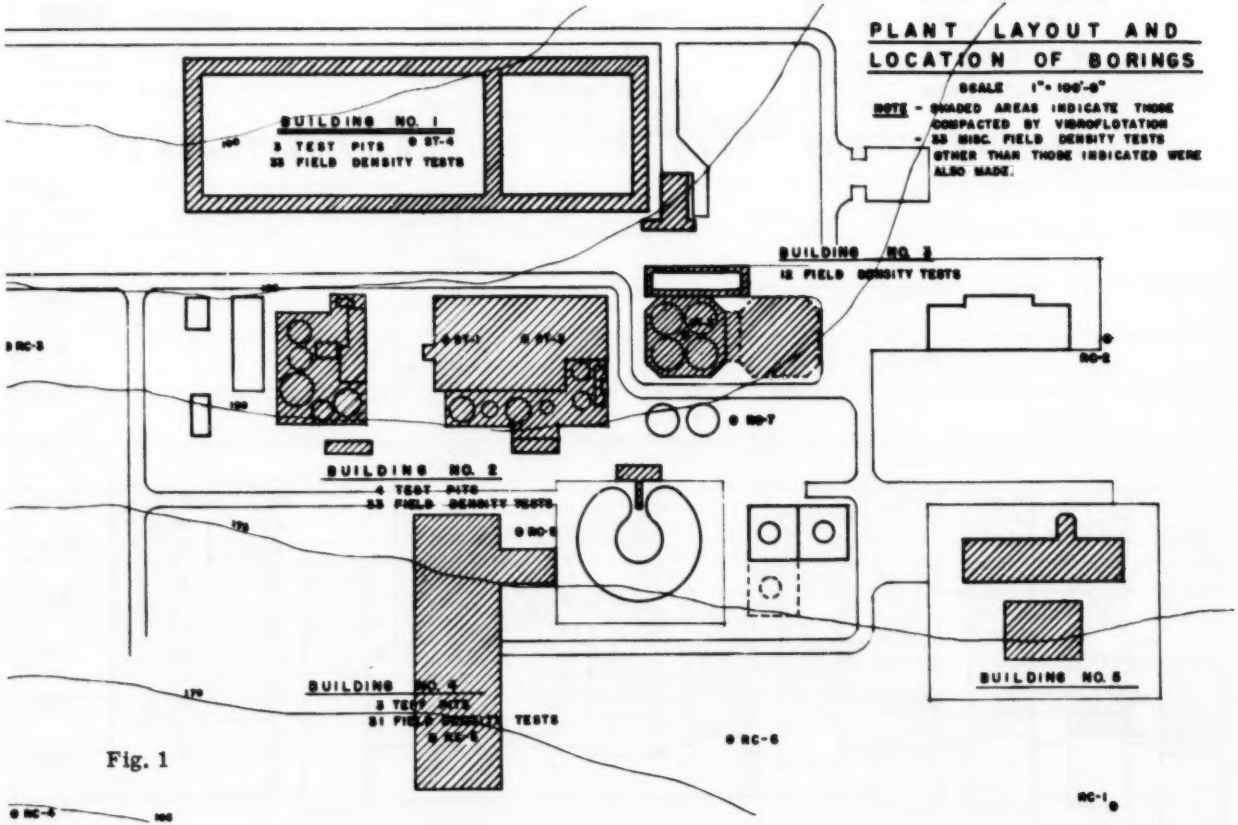

PLANT LAYOUT AND
LOCATION OF BORINGS
SCALE 1"= 100'-0"
NOTE - SHADED AREAS INDICATE THOSE COMPACTED BY VIBROFLOTATION
- 33 MISC. FIELD DENSITY TESTS OTHER THAN THOSE INDICATED WERE ALSO MADE.
BUILDING NO. 1
3 TEST PITS
33 FIELD DENSITY TESTS
BUILDING NO. 3
12 FIELD DENSITY TESTS
BUILDING NO. 2
4 TEST PITS
33 FIELD DENSITY TESTS
BUILDING NO. 4
3 TEST PITS
31 FIELD DENSITY TESTS
BUILDING NO. 5
RC-1
RC-2
RC-3
RC-4
RC-5
RC-6
RC-7
ST-1
ST-2
ST-4
100
120
170

Fig. 1

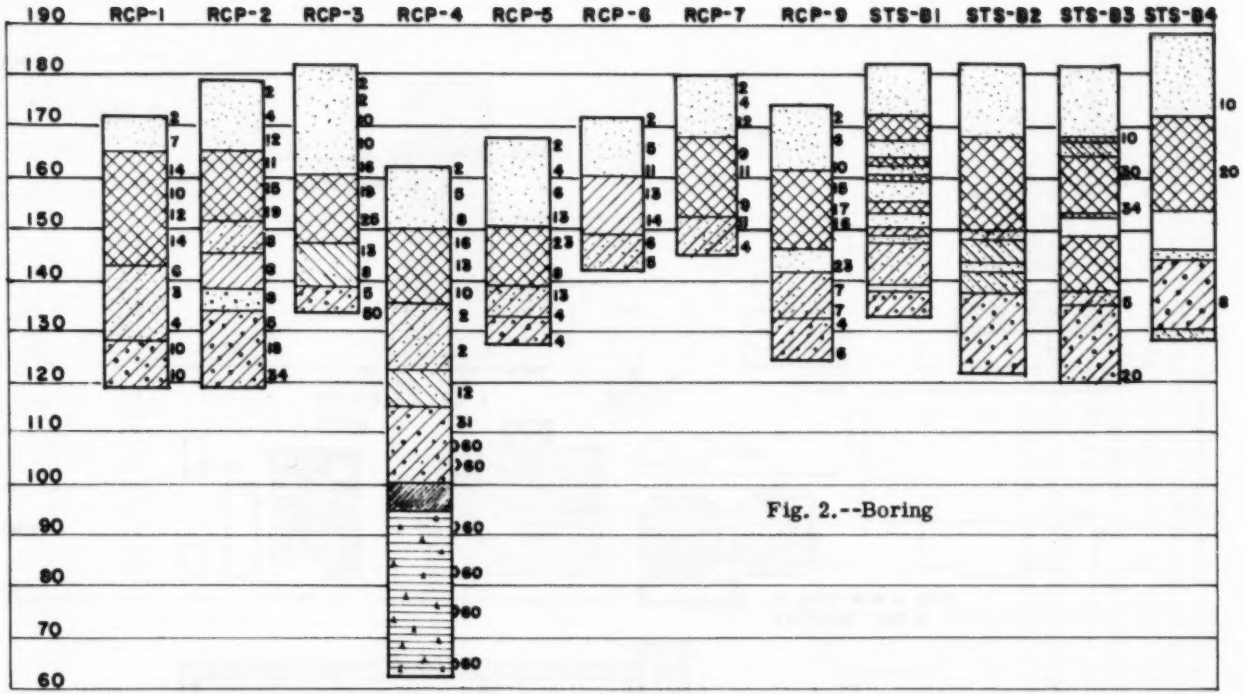
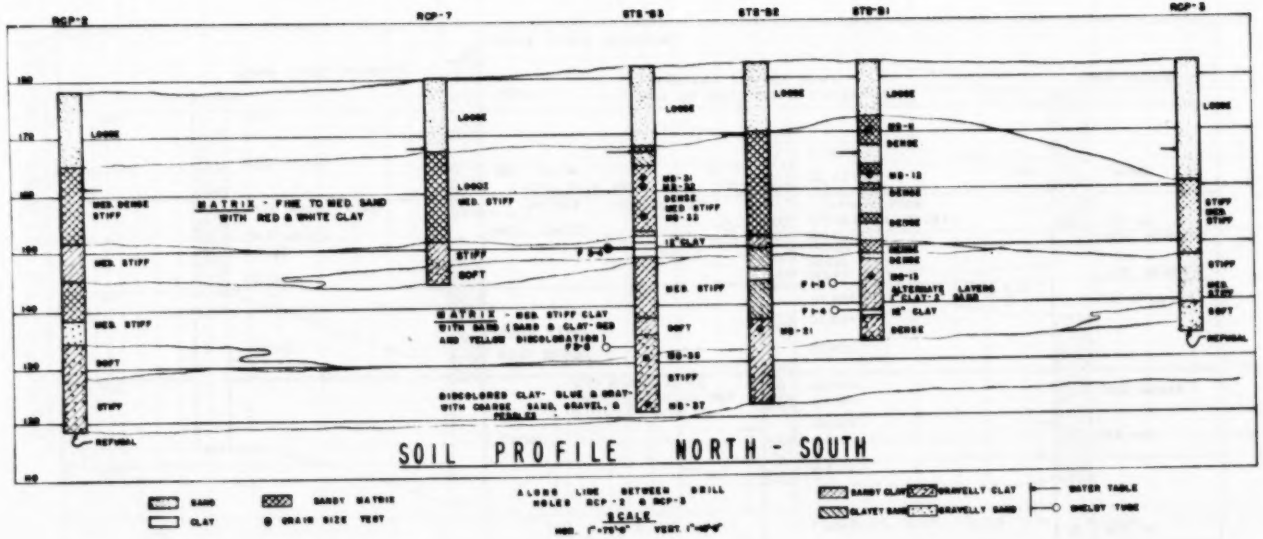
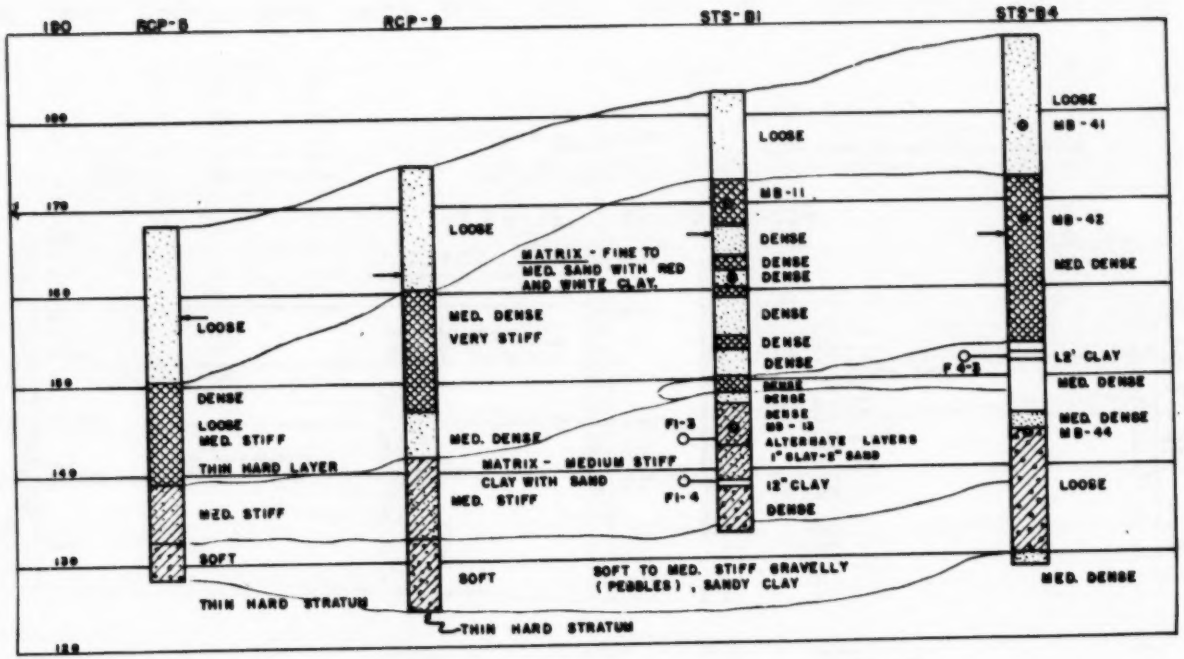

190
180
170
160
150
140
130
120
110
100
90
80
70
60
RCP-1
RCP-2
RCP-3
RCP-4
RCP-5
RCP-6
RCP-7
RCP-9
STS-B1
STS-B2
STS-B3
STS-B4
GENERALIZED BORING LOG DATA AND STANDARD PENETRATION RESULTS
SAND
CLAY
BOULDER
SANDY MATRIX
SANDY CLAY
CLAYEY SAND
GRAVELLY SAND
GRAVELLY CLAY
SANDY CLAY WITH BOULDERS

Fig. 2.--Boring

Fig. 3.--Soil Profile North-South





SOIL PROFILE EAST - WEST

ALONG LINE BETWEEN DRILL HOLES RCP-8 & STS-B4

-SCALE-

HOR. 1" = 75'-0" VERT. 1" = 10'-0"

Fig. 4

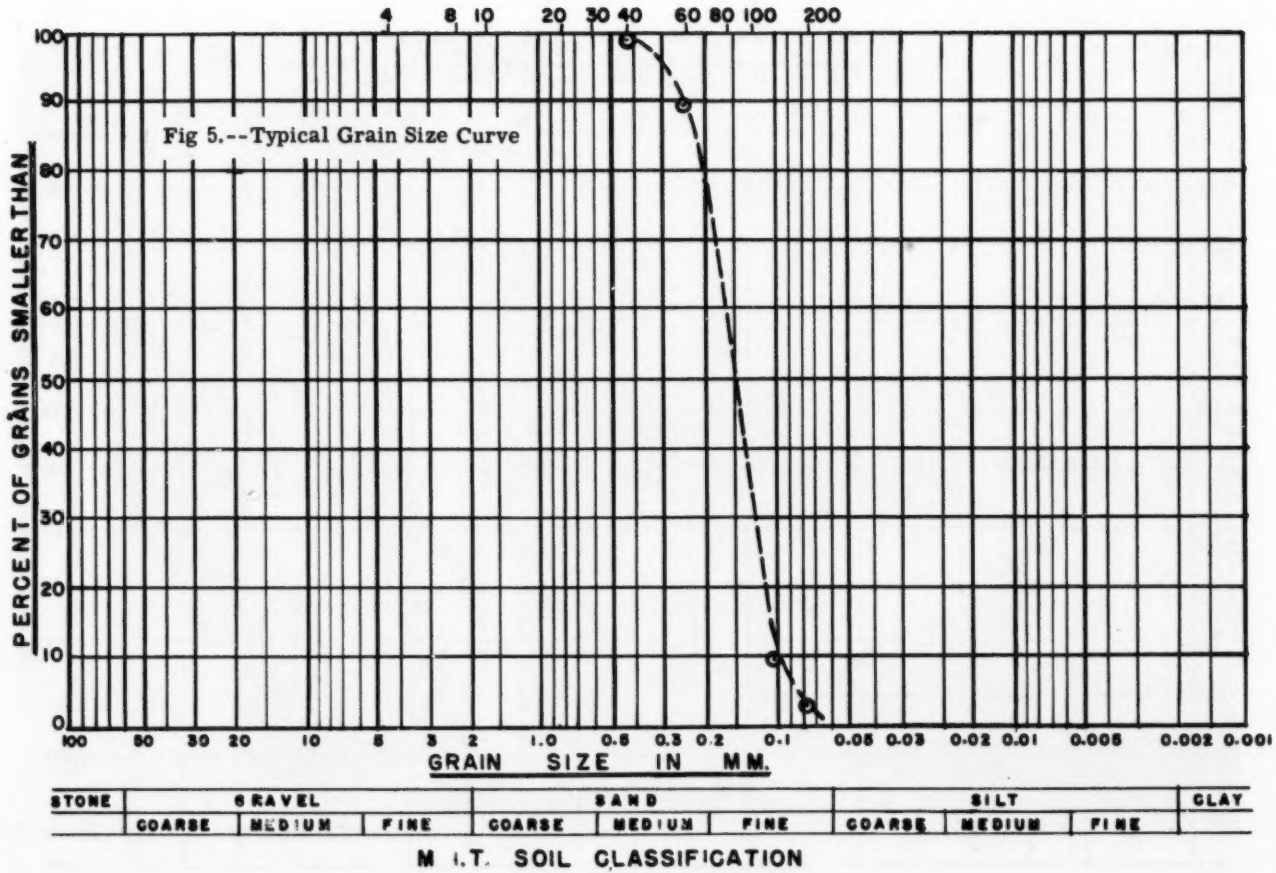


Fig 5.--Typical Grain Size Curve

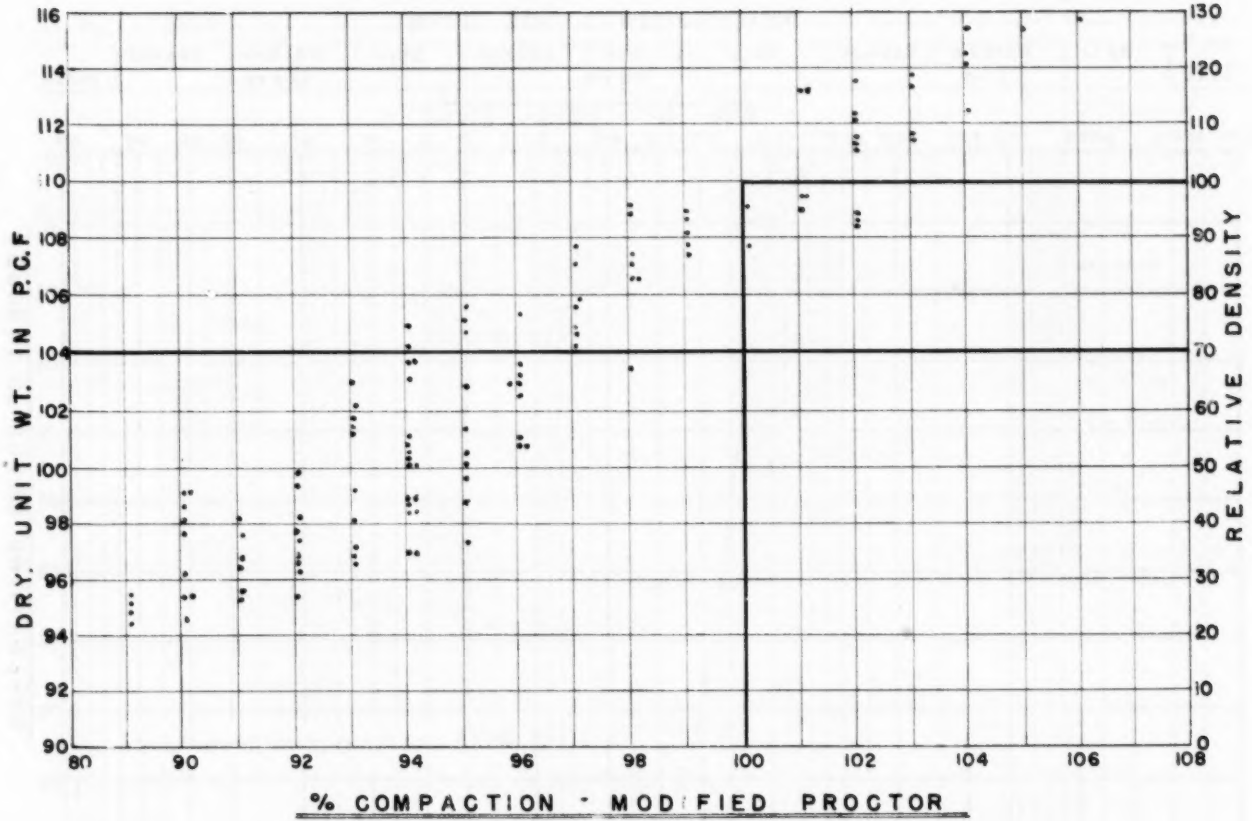


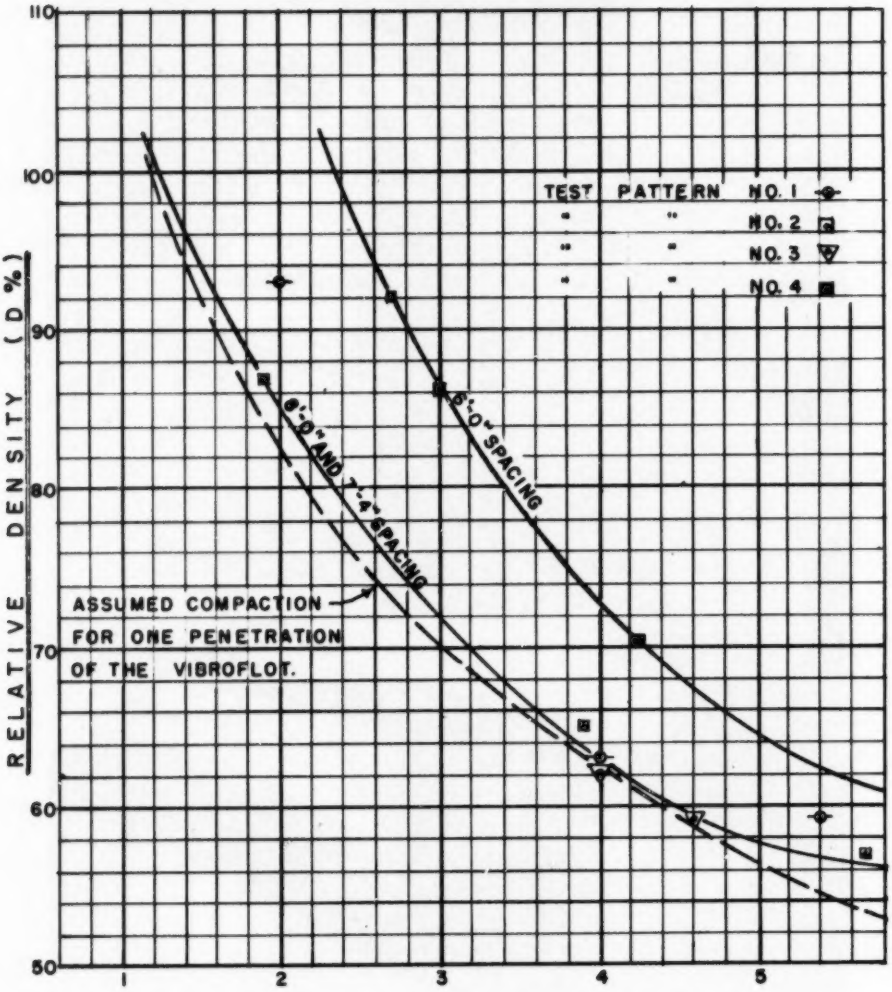
Fig. 6. COMPACTION RESULTS FROM EGLIN FIELD

Fig. 7. Apparatus for Field Density Tests

CURVE FOR

RELATIVE DENSITY - DISTANCE

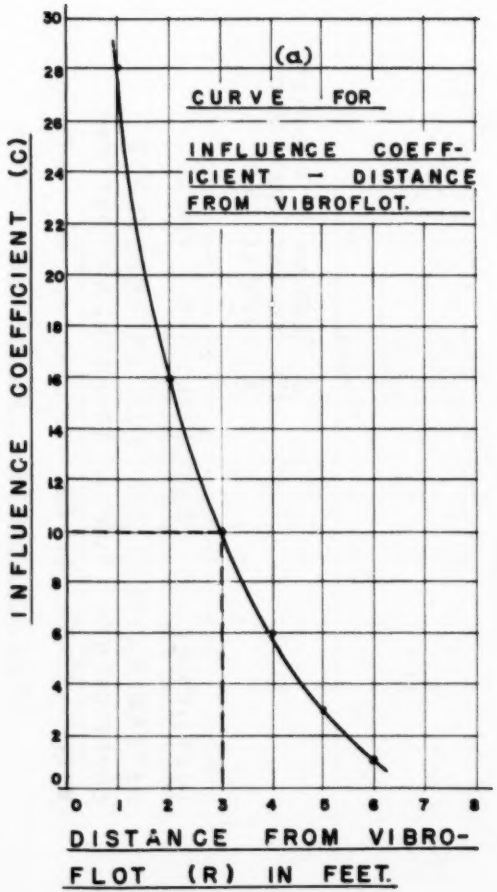
FROM VIBROFLOT.



DISTANCE FROM VIBROFLOT (R)

IN FEET

Fig. 8.


(a)
CURVE FOR
INFLUENCE COEFFICIENT - DISTANCE FROM VIBROFLOT.
INFLUENCE COEFFICIENT (C)
0 1 2 3 4 5 6 7 8
DISTANCE FROM VIBROFLOT (R) IN FEET.

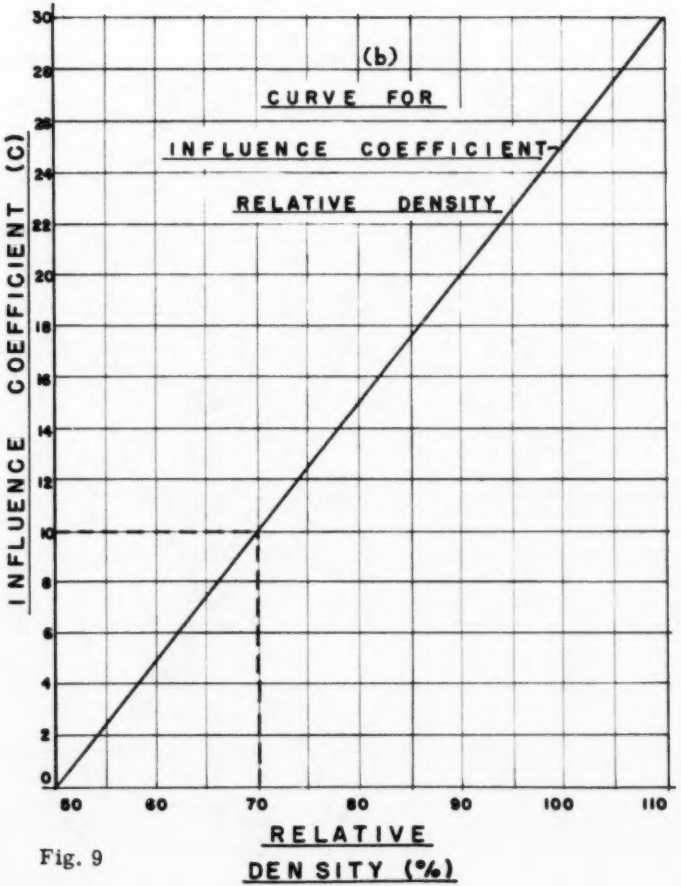
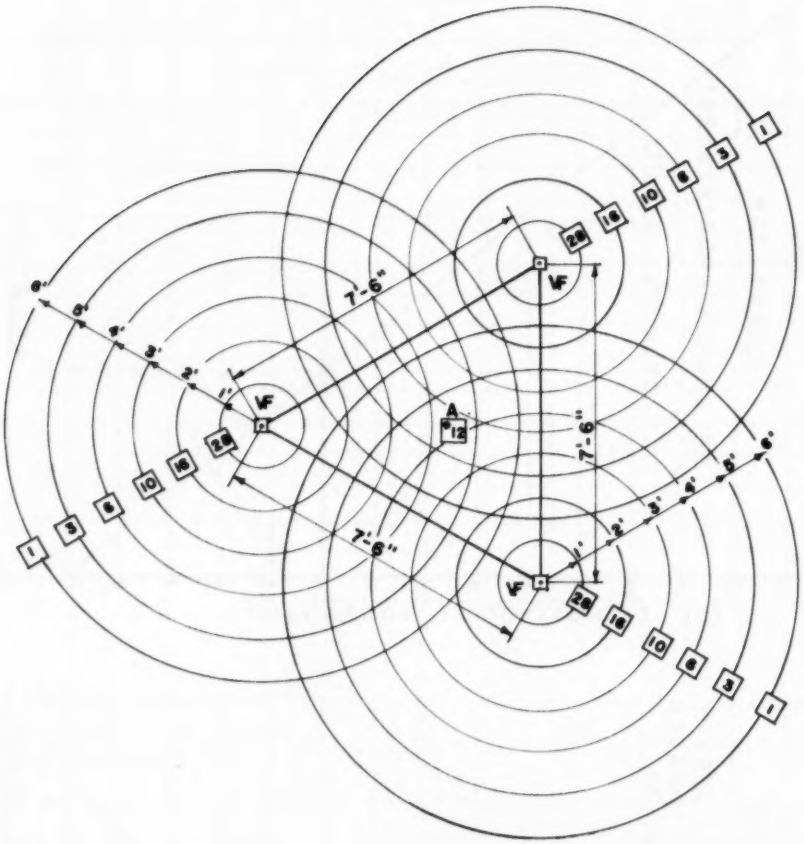

(b)
CURVE FOR
INFLUENCE COEFFICIENT - RELATIVE DENSITY
INFLUENCE COEFFICIENT (C)
50 60 70 80 90 100 110
RELATIVE DENSITY (%)

Fig. 9

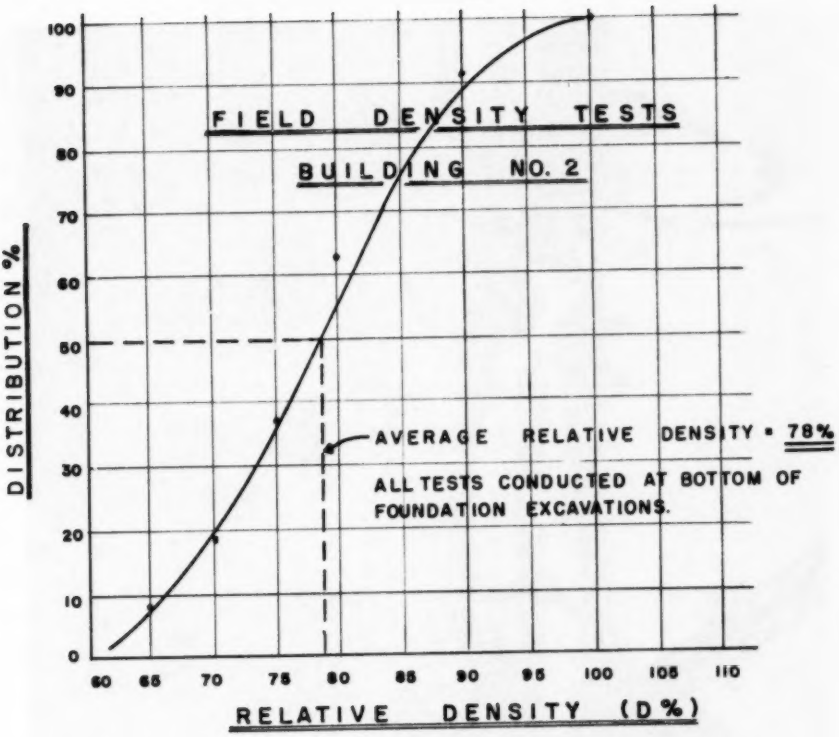


VF - POSITION OF VIBROFLOT
□ - INFLUENCE COEFFICIENT
A - CRITICAL POINT

INFLUENCE PATTERN FOR

DETERMINATION OF SUM OF INFLUENCE COEFFICIENTS FOR A TRIANGULAR PATTERN WITH 7'-6" SIDES.

Fig. 10


FIELD DENSITY TESTS
BUILDING NO. 2
DISTRIBUTION %
AVERAGE RELATIVE DENSITY = 78%
ALL TESTS CONDUCTED AT BOTTOM OF FOUNDATION EXCAVATIONS.
RELATIVE DENSITY (D%)

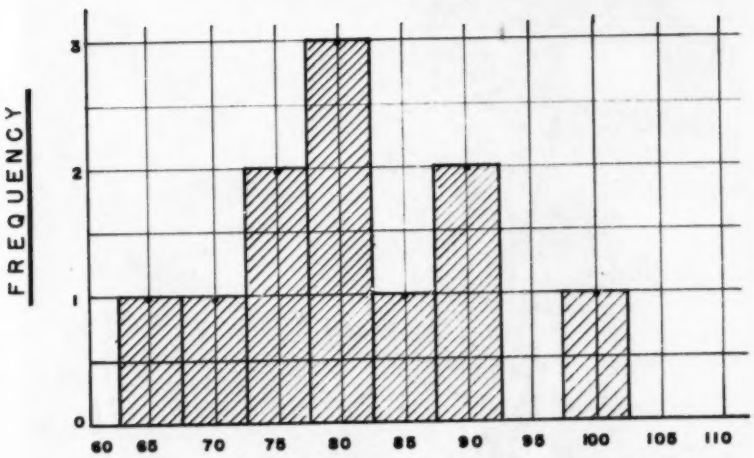

FREQUENCY
RELATIVE DENSITY (D%)

Fig. 11

Fig. 12. Excavation in Compacted Area

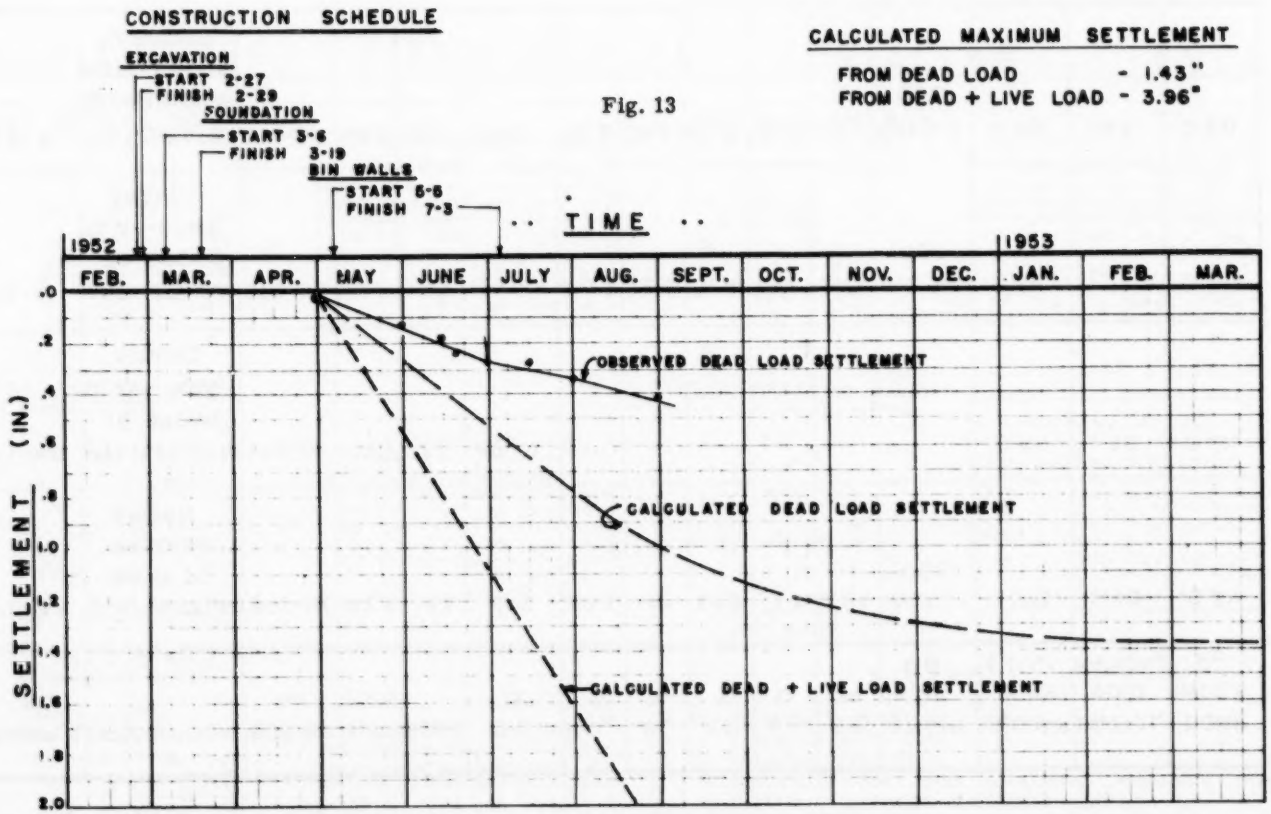


Fig. 13

SETTLEMENT TIME CURVE
FOR SILO STORAGE BUILDING

Table 1

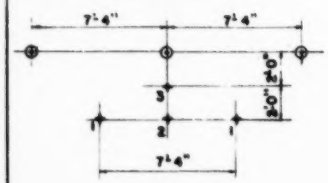
SOIL DATA

SAMPLE NO.	DESCRIPTION	BORING NO.	SURF. ELEV.	ELEV.	M.C. %	L.L. %	P.L. %	P.I. %	S.L. %	S.P. GR.	GRAIN SIZE MM.	UNCONF. COMP. q_u (T/ft^2)	INITIAL VOID RATIO (e_o)	COMP. INDEX C_c
F1-3A	CLAY MOTTLED GRAY TO REDDISH BROWN	STS-B1	182.6	143	51.9	53.3	14.1	39.3	17.3	2.84	100% < .002	0.083	1.48	0.41
F1-4B	MOTTLED GRAY TO BROWN CLAY - SOME GRAVEL	STS-B1	182.6	137	42.10	-	-	-	-	-		0.44	1.36	0.21
F3-4	MOTTLED GRAY TO BROWN CLAY-SOME GRAVEL	STS-B3	181.9	150	54.80	80.3	28.9	51.4	18.1	2.73	100% < .002	0.59	1.71	0.22
F4-3	ALTERNATE SEAMS of CLAY MOTTLED GRAY TO BROWN	STS-B4	189.5	152	68.0	83.8	38.6	45.2	19.0	-	100% < 002	0.32	1.62	0.20

RESULTS OF VIBROFLOTATION COMPACTION TESTS

TEST PATTERN NO. 1

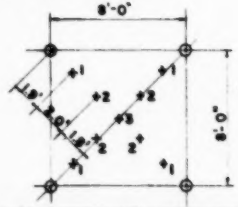
7'-4" △ LINE PATTERN



AVERAGE VALUES					
DEPTH	POINT	D(%)	R(FT.)	S(FT.)	C
AVG. OF 4, 6, 8 FT.	1	59	5.4	7.33	4
	2	63	4.0	7.33	6
	3	93	2.0	7.33	16

TEST PATTERN NO. 2

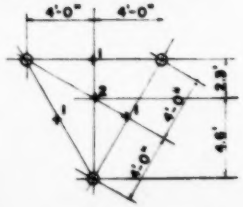
8'-0" SQUARE PATTERN



AVERAGE VALUES					
DEPTH	POINT	D(%)	R(FT.)	S(FT.)	C
AVG. OF 4, 6, 8 FT.	1	87	1.9	8.0	16
	2	66	3.9	8.0	9
	3	57	5.7	8.0	4

TEST PATTERN NO. 3

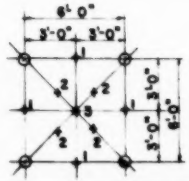
8'-0" △ TRIANGULAR PATTERN



AVERAGE VALUES					
DEPTH	POINT	D(%)	R(FT.)	S(FT.)	C
AVG. OF 4, 6, 8 FT.	1	62	4	8.0	12
	2	59	4.6	8.0	10

TEST PATTERN NO. 4

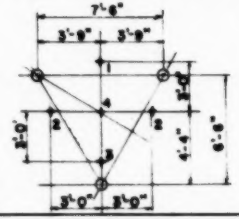
6'-0" △ SQUARE PATTERN



AVERAGE VALUES					
DEPTH	POINT	D(%)	R(FT.)	S(FT.)	C
AVG. OF 4, 6, 8 FT.	1	86	3	6	20
	2	92	2.75	6	22
	3	70	4.25	6	18

TEST PATTERN NO. 5

7'-6" TRIANGULAR PATTERN



AVERAGE VALUES					
DEPTH	POINT	D(%)	R(FT.)	S(FT.)	C
AVG. OF 3, 6, 9 FT.	1	80	3.84	7'-6"	18
	2	73	2.28	7'-6"	14
	3	88	1.33	7'-6"	26
	4	68	4.34	7'-6"	12

LEGEND

- ⊙ VIBROFLOT HOLE.
- \+ FIELD DENSITY TEST.
- D RELATIVE DENSITY.
- R DISTANCE FROM POINT TO NEAREST VIBROFLOT HOLE.
- S DISTANCE BETWEEN VIBROFLOT HOLES.
- C SUM OF INFLUENCE COEFFICIENTS.

TABLES 2 TO 6

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